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MATHEMATICAL MODEL STUDY OF A FLOW CONTROL PLAN FOR THE CHESAPEAKE AND DELAWARE CANAL

by

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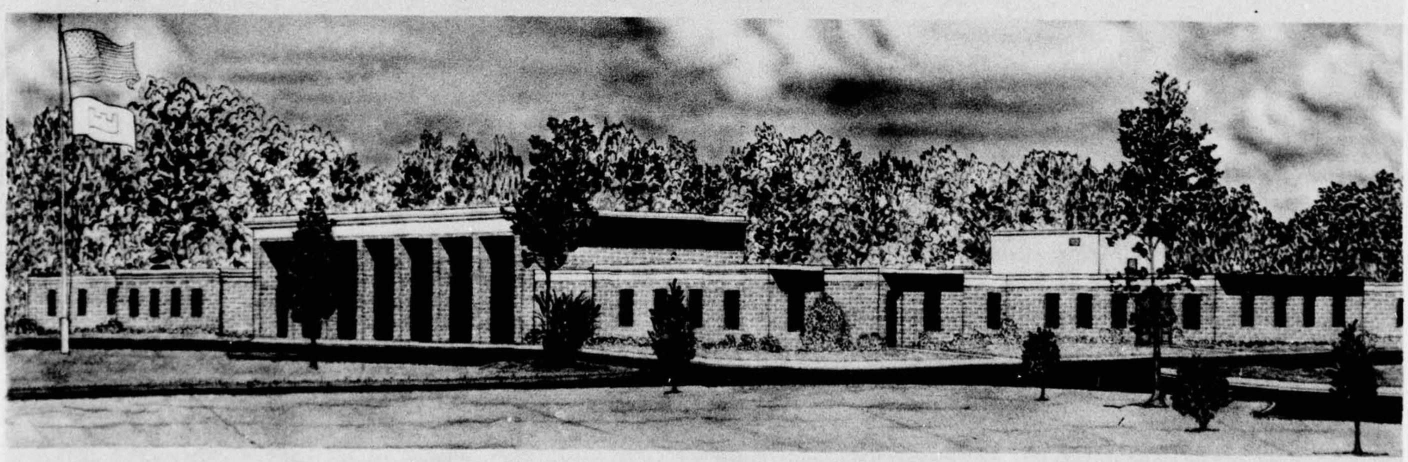

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In 1954 Congress authorized an enlargement of the sea level canal connecting the Delaware River and the Chesapeake Bay. This enlargement was to provide a 35-ft by 450-ft channel section instead of the 27-ft by 250-ft channel which existed at that time. As of 1974, about 90 percent of the project has been physically completed with about 1 mile of the eastern end of the canal remaining to be dredged. Extensive studies to define the ecological effects of channel enlargement have not thus far indicated any significant adverse
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20. ABSTRACT (Continued).

effect which would warrant flow control in the canal. However, it was considered advisable to investigate the feasibility of flow control schemes in case such control ever became desirable. The primary objective of a flow control plan would be to restore net flows to approximately preproject conditions (i.e., 250-ft by 27-ft canal conditions). A second requirement suggested by the ecological studies was that the velocity in the main canal should not be allowed to fall below 1 fps during the majority of the tidal cycle. This requirement stems from the fact that during the spawning season, striped bass eggs (which cannot survive on the channel bottom) rely upon flow velocities to maintain their suspension in the water column. Of the various flow control schemes considered (see Appendix A), a navigation lock and dam in the main canal, along with a small bypass canal, was selected as perhaps the best flow control plan to satisfy simultaneously the two criteria previously discussed. A mathematical model, called SOCHMJ, for the computation of unsteady flows in a system composed of an unlimited number of open channels has been employed to determine the feasibility of such a flow control plan. Several test cases employing different bypass channel sections and lengths were run. During these tests it was found that the velocity in the bypass canal became so large for some bypass canal lengths as to become hazardous to small craft navigating through the bypass. Therefore, the additional criterion of restricting velocities in the bypass canal to be less than 6.0 fps was imposed. The results from the mathematical model study indicate that the proposed flow control plan is definitely feasible. Furthermore, it is shown that during extreme tide conditions a bypass channel section of 18 ft by 200 ft and a length of 1.0 mile is almost sufficient to satisfy simultaneously the net flow criterion, the main canal velocity criterion, and the bypass canal velocity criterion. Further refinement of this flow control plan would of course be required if construction of such a plan became desirable.

PREFACE

The study reported herein was conducted at the U. S. Army Engineer Waterways Experiment Station (WES) during the period March 1974 - May 1974. It was sponsored by the U. S. Army Engineer District, Philadelphia.

Dr. B. H. Johnson, Mathematical Hydraulics Division, conducted the study and prepared this report under the general supervision of Messrs. H. B. Simmons, Chief of the Hydraulics Laboratory, and M. B. Boyd, Chief of the Mathematical Hydraulics Division. Mr. J. C. Smith, Mathematical Hydraulics Division, assisted in the study. Mr. T. C. Hill, Estuaries Division, conducted the physical model test, the results of which aided the mathematical model study.

Director of WES during the conduct of this study and the preparation of this report was COL G. H. Hilt, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	meters
feet per second	0.3048	meters per second
cubic feet	0.02831685	cubic meters
cubic feet per second	0.02831685	cubic meters per second
miles (U. S. statute)	1.609344	kilometers

PART I: INTRODUCTION

Background

1. A sea level canal connecting the Delaware River and the Chesapeake Bay (C & D Canal) was completed in 1927 with a channel width of 90 ft* and a depth of 12 ft. In 1935 Congress authorized enlarging the channel to a 27-ft depth by 250-ft width. This work was completed in 1954. A further modification, authorized by Congress in 1954, provided for a 35-ft-deep, 450-ft-wide channel. Construction was started in 1956. Completion was originally scheduled for 1969; however, due to initial budgetary limitations plus a delay in the completion of a hydrographic and ecological study, the presently scheduled completion date is 1974. At the present time all dredging has been completed except the enlargement of about 1 mile of the canal at its eastern end.

2. The hydrographic and ecological study has been conducted by the Chesapeake Biological Laboratory, University of Maryland; the Chesapeake Bay Institute, John Hopkins University; and the College of Marine Studies, University of Delaware.[1] Simultaneously, the Waterways Experiment Station, (WES) Vicksburg, Mississippi, conducted physical and mathematical model studies to determine the hydrographic conditions of the canal after completion.[2] Results of these studies have not to date indicated any significant adverse effect which would warrant flow control in the canal. However, it was considered advisable to investigate the feasibility of flow control schemes in case such control ever became desirable. The primary objective of a flow control plan would be to return net flows to approximately preproject conditions (i.e., 250-ft by 27-ft canal conditions).

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

A second requirement suggested by the ecological studies was that the velocity in the main canal should not be allowed to fall below 1 fps during the majority of the tidal cycle. This requirement stems from the fact that during the spawning season, striped bass eggs (which cannot survive on the bottom) rely upon flow velocities to maintain their suspension in the water column.

3. With the purpose of arriving at a suitable solution for reducing the net flow in the canal to values comparable to those encountered with a 27-ft by 250-ft channel, personnel from the Philadelphia District and the North Atlantic Division met on 17 January 1974 at WES to discuss possible methods of solution with WES personnel. At this meeting it was agreed that the most logical flow control scheme was a navigation lock and dam in the canal with a smaller bypass canal. To minimize excavation the bypass canal should be located in the reach between St. Georges Bridge and Reedy Point Bridge and to the south of the main canal. Such a scheme is illustrated in fig. 1. A brief discussion on alternate flow control plans and the reasons for selecting the scheme discussed above is presented in Appendix A.

Purpose and Scope

4. On 12 February 1974 the Mathematical Hydraulics Division at WES (MHD) proposed to the Philadelphia District a mathematical model study of the flow control plan outlined above to determine bypass canal lengths and cross sections that would be required to reduce the net flow or maximum velocity in the main canal to preproject conditions. At that time, it was noted that the first few tests with the math model would be based on the assumption that the above flow control plan would not

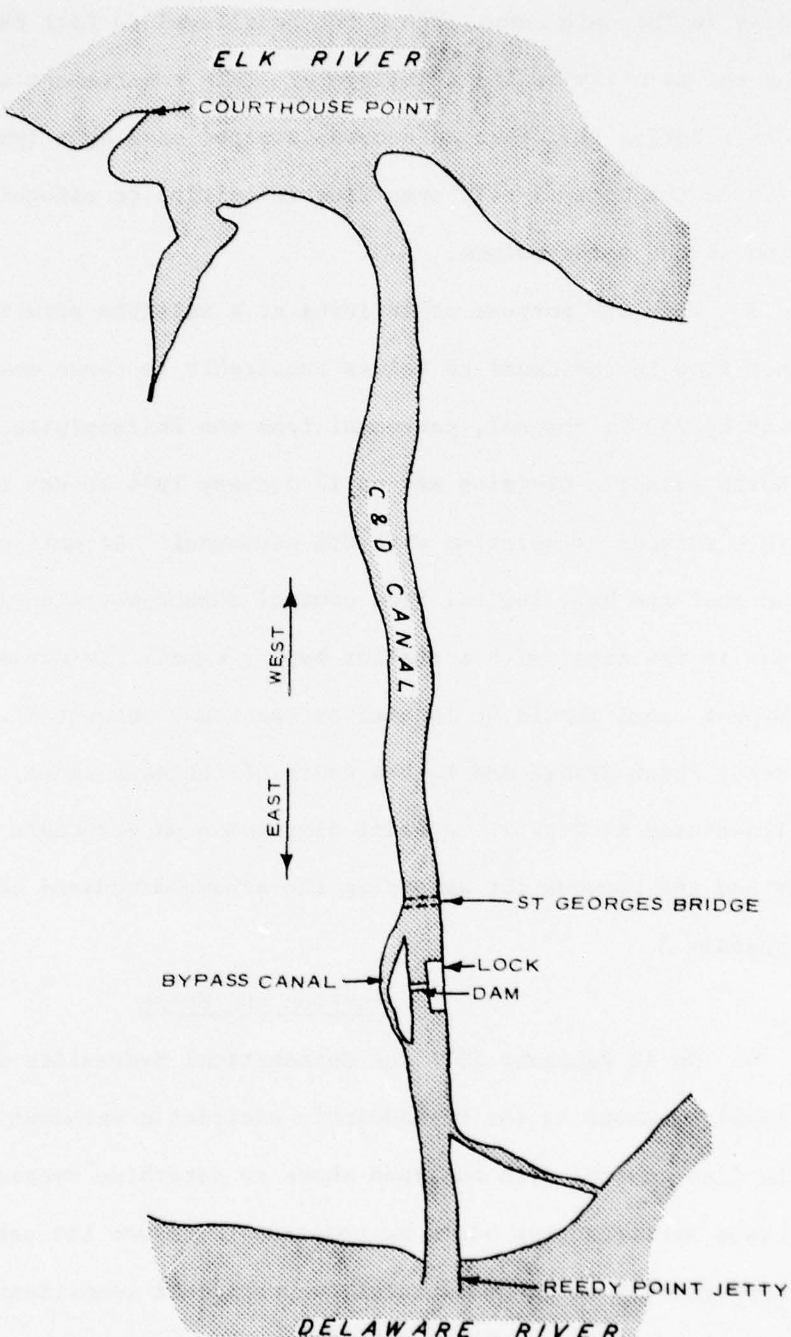

Fig. 1. Proposed flow control scheme

significantly alter the tide curves at Reedy Point and at Courthouse Point. After a few exploratory computations with the math model, a test in the physical model would be run to determine the accuracy of the assumption.

PART II: MATHEMATICAL MODEL FMPLOYED

5. A computer program called SOCHMJ which utilizes Stoker's explicit centered finite difference scheme to provide numerical solutions of the differential equations governing unsteady flow in open channels has been developed by the MHD.[3] These equations, which are often referred to as the equations of St. Venant, are statements of the conservation of mass and momentum of the flow field and may be written as:*

Continuity: $$\frac{\partial h}{\partial t} + \frac{1}{B}\frac{\partial (AV)}{\partial x} - \frac{q}{B} = 0$$

Momentum: $$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial h}{\partial x} + \frac{qV}{A} + \frac{gn^2 V|V|}{2.21R^{4/3}} = 0$$

Since an explicit scheme is utilized, the stability criterion

$$\left(V + \sqrt{g\frac{A}{B}}\right)\frac{\Delta t}{\Delta x} \leq 1 - \frac{gn^2|V|\Delta t}{2.21R^{4/3}}$$

must be satisfied by the time and spatial steps, i.e., Δt and Δx, to ensure a stable solution of the difference equations that converges to the solution of the differential equations. SOCHMJ provides the capability of modeling a system containing an unlimited number of junctions and split channels.

6. Data required for the operation of SOCHMJ are read from cards. The first data card contains basic information such as the total number of net points in the system, the total number of junctions and branches, and the time step employed in the computations. The second data group contains information about each branch. Such information consists of the

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* For convenience, symbols are listed and defined in the Notation (Appendix B).

numbers of the first and last net points of each branch, the type of boundary conditions prescribed at the various boundaries, and the size of the spatial step to be employed for each branch. The third data group contains information about the junctions in the system being modeled such as the numbers of the branches coming into each junction. The next major data group consists of the tables of geometric data. A table of top width, flow area, (hydraulic radius)$^{2/3}$ and Manning's "n" versus elevation must be input at each net point in the system. These tables were obtained from a previous math modeling effort on the C & D Canal performed by Boyd.[2] The fifth data group specifies initial values of the elevation and discharge at all grid points on the first two time lines. These are required in order to initiate the computations. The nature of the governing equations is such that the effects of initial conditions are not felt after a sufficient period of time. For the test cases of the study described herein, three complete tidal cycles were run, with the results presented taken from the third cycle. This ensures that the results are independent of the initial conditions. The final major data group required by SOCHMJ consists of the time-dependent boundary conditions which must be prescribed at each open boundary. At such a boundary, elevations, discharges, or a rating curve may be prescribed as the boundary condition.

PART III: APPLICATION OF SOCHMJ TO THE PROPOSED FLOW CONTROL PLAN

7. A plan view of the C & D Canal and the proposed flow control scheme is presented in fig. 1. In order to apply SOCHMJ to such a physical system, one must represent the system by various junctions and the corresponding branches composing each junction. This representation is illustrated in fig. 2. From fig. 2 it can be seen that the system is composed of five branches and two junctions. The first junction is fixed at St. Georges Bridge, whereas, the location of the second junction of the bypass canal and the main canal varies depending upon the length of the bypass. The branches composing the system are as labeled on fig. 2.

8. As previously discussed, the spatial step size for each branch must be input. The Δx employed for branch 1 was 2112 ft and remained the same for all tests. Due to the fact that the length of the bypass canal was varied, the spatial steps used for the other four branches also varied. These values, for different bypass lengths, are presented in table 1 along with the corresponding time step utilized for the overall system in each case. Remember that when the spatial step on a branch is changed, quite often the time step for the system must also be changed to maintain the stability criterion.

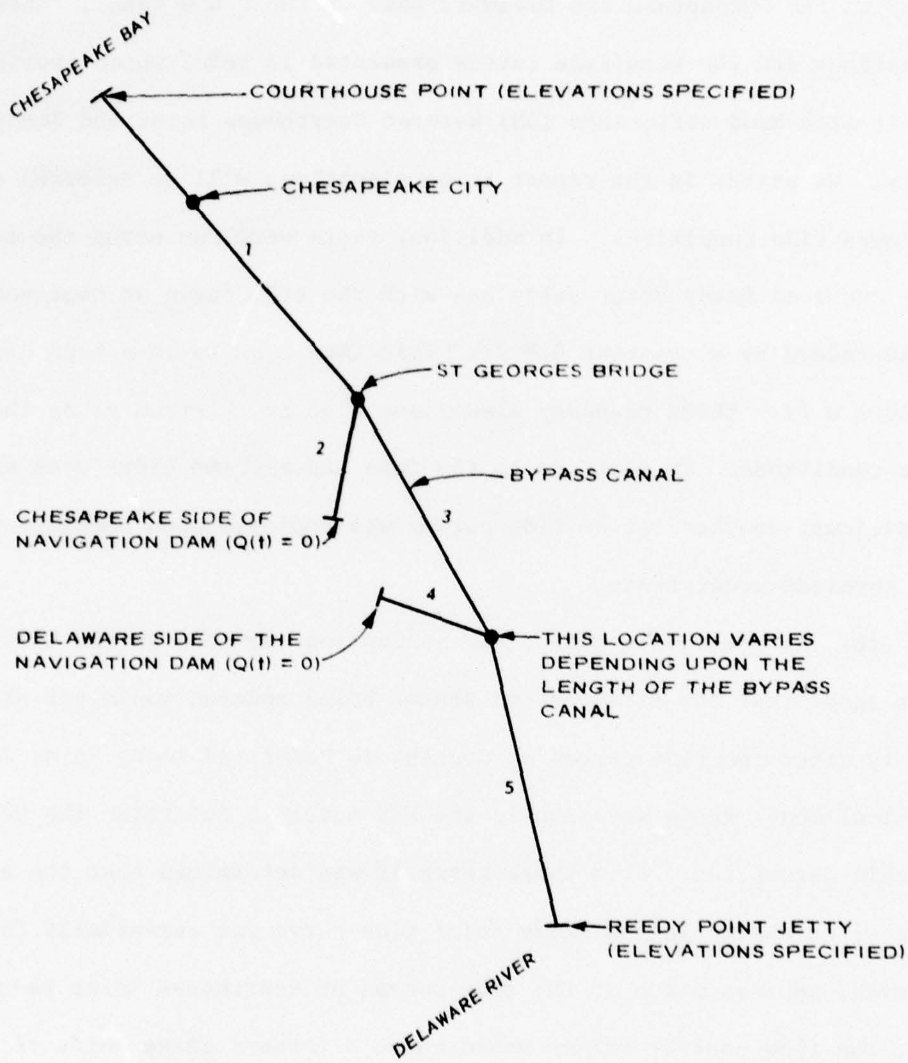

Fig. 2. Representation of physical system

9. As indicated on fig. 2, elevations as functions of time were prescribed as boundary conditions at Courthouse Point and Reedy Point Jetty on the Chesapeake and Delaware ends of the C & D Canal. These elevations are the mean tide curves presented in ref. 2 which provide a 0.2-ft mean head difference (ΔH) between Courthouse Point and Reedy Point Jetty. Hereafter in the report these elevations will be referred to as the mean tide conditions. In addition, tests were run using the same tide curve at Reedy Point Jetty but with the tide curve at Courthouse Point raised by a constant 0.8 ft. This then results in a head differential of ΔH=1.0 ft. These boundary elevations will be referred to as the extreme tide conditions. In addition to the mean and extreme tides used as boundary conditions, another set of tide curves was employed upon completion of the physical model tests.

10. As previously noted, the assumption was made in the initial test cases that the flow control scheme being modeled would not sifnificantly alter the tide curves at Courthouse Point and Reedy Point Jetty. Physical model tests were run in the WES model to determine the accuracy of this assumption. From these tests it was determined that the assumption of no change in the Reedy Point tide curve was essentially correct. However, an inspection of the tide curves at Courthouse Point revealed that the flow control scheme would cause a forward phase shift of about 30 minutes. Results from the physical model tests leading to the above conclusions are presented in fig. 3. It should be noted that the physical model tests were run for the case of a bypass canal with a length of 1.7 miles and a cross section of 15 ft by 150 ft. However, computed

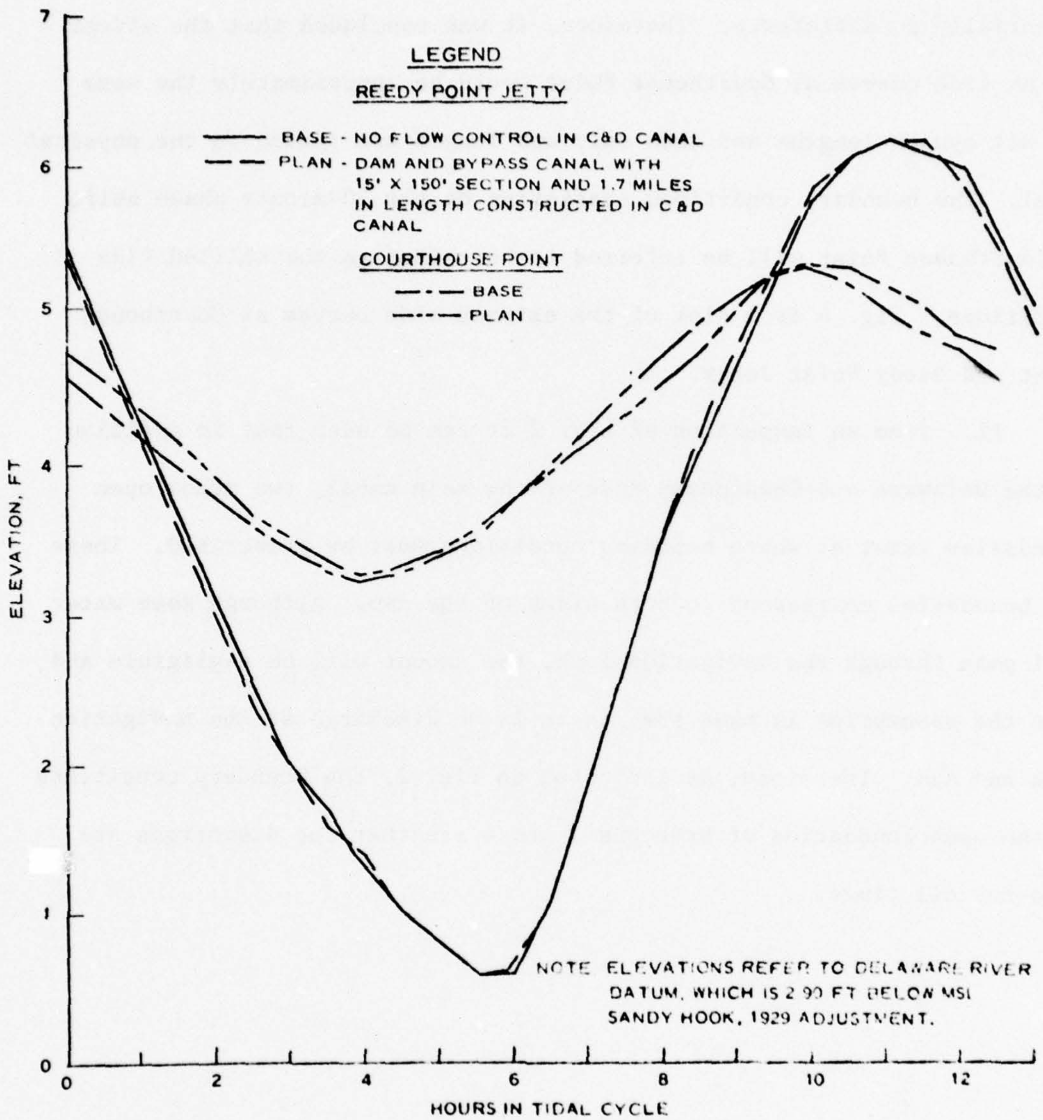

Fig. 3. Tide curves from physical model tests

elevations at Chesapeake City for different bypass lengths revealed essentially no difference. Therefore, it was concluded that the effect on the tide curves at Courthouse Point would be approximately the same for all bypass lengths and thus only one length was tested in the physical model. The boundary conditions consisting of the 30-minute phase shift at Courthouse Point will be referred to hereafter as the shifted tide conditions. Fig. 4 is a plot of the extreme tide curves at Courthouse Point and Reedy Point Jetty.

11. From an inspection of fig. 2 it can be seen that in addition to the Delaware and Chesapeake ends of the main canal, two other open boundaries exist at which boundary conditions must be prescribed. These two boundaries correspond to both sides of the dam. Although some water will pass through the navigation lock, the amount will be negligible and thus the assumption is made that there is no discharge at the navigation lock and dam. Therefore, as indicated on fig. 2, the boundary conditions at the open boundaries of branches 2 and 4 are that the discharges are zero for all times.

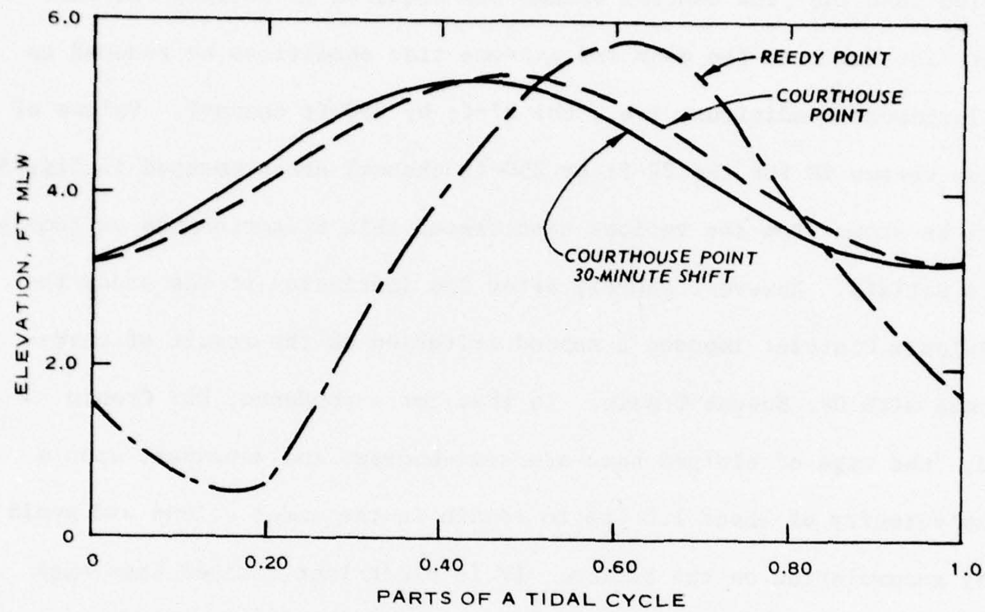

Fig. 4. Boundary extreme tide curves

PART IV: PRESENTATION OF RESULTS

12. In the planning phase of the study described herein, the only criterion that the flow control scheme was required to satisfy was that the net flow for both the mean and extreme tide conditions be reduced to pre-enlargement conditions, i.e., the 27-ft by 250-ft channel. Values of net flow versus ΔH for the 27-ft by 250-ft channel are presented in fig. 5. As will be shown from the various test cases, this criterion was extremely easy to satisfy. However, shortly after the initiation of the study the Philadelphia District imposed a second criterion as the result of correspondence with Dr. Eugene Cronin. In that correspondence, Dr. Cronin stated, "the eggs of striped bass are semi-buoyant and dependent upon a minimum velocity of about 1.0 fps to remain in the water column and avoid loss by accumulation on the bottom. It is clear that striped bass eggs survive normal tidal slacks in the canal, which are of exceptionally short duration. However, longer periods of low velocity are undesirable and the flow control structure should not cause them in the canal." Fig. 6 is a plot of velocities of Chesapeake City, taken from ref. 2, for the 27-ft by 250-ft channel. Note that the velocity remains above 1.0 fps for approximately 70 percent of the tidal cycle. In the attempt to force the flow control scheme to satisfy both of the above criteria, several test cases employing different bypass canal lengths and cross sections were run. These various tests are listed below.

TEST	BYPASS LENGTH	BYPASS SECTION	TIDE CONDITIONS
1	3.0 miles	15' x 150'	Mean
2	3.0 miles	15' x 150'	Extreme

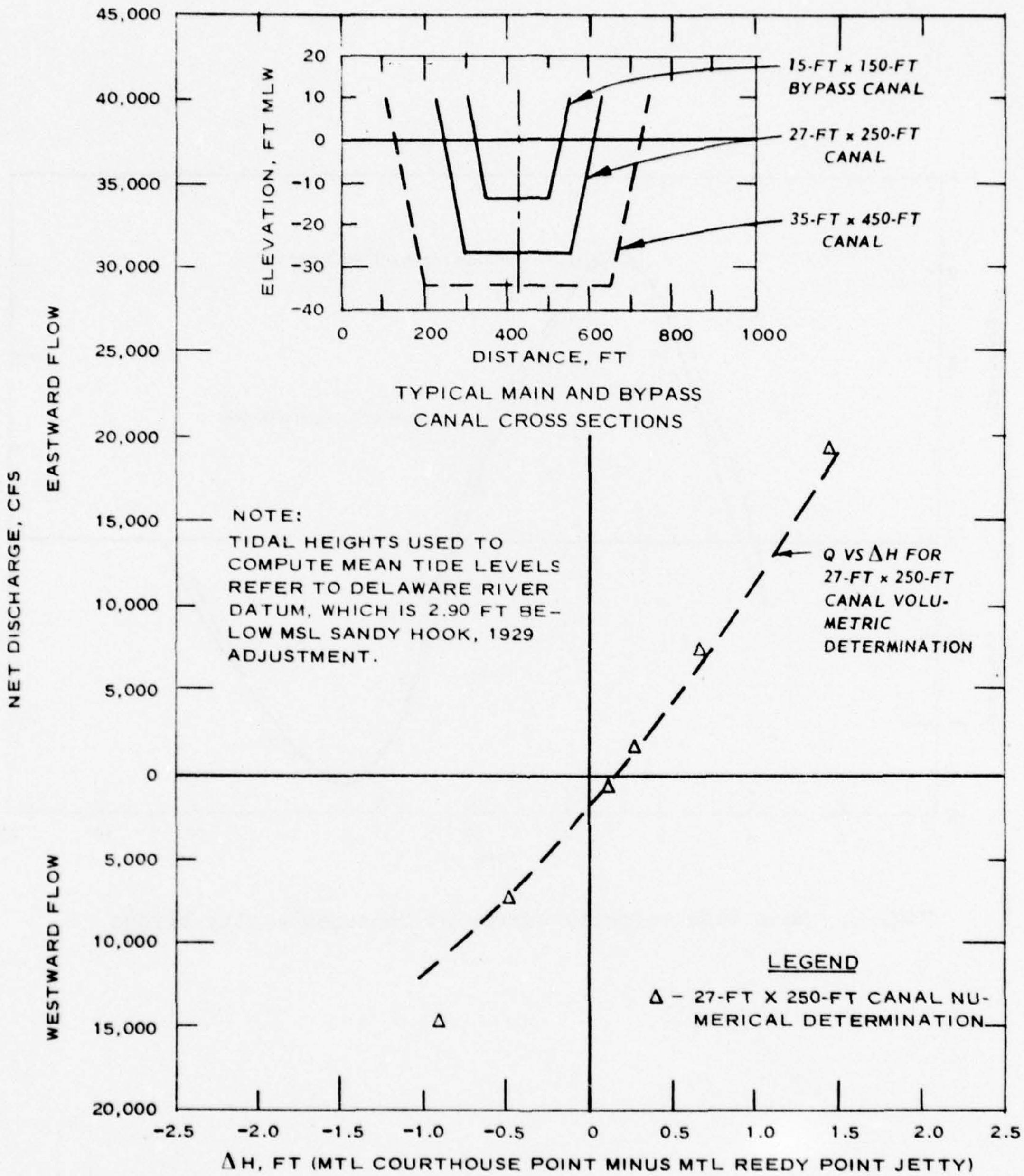

Fig. 5. Comparison of volumetric and numerical determination of net discharge

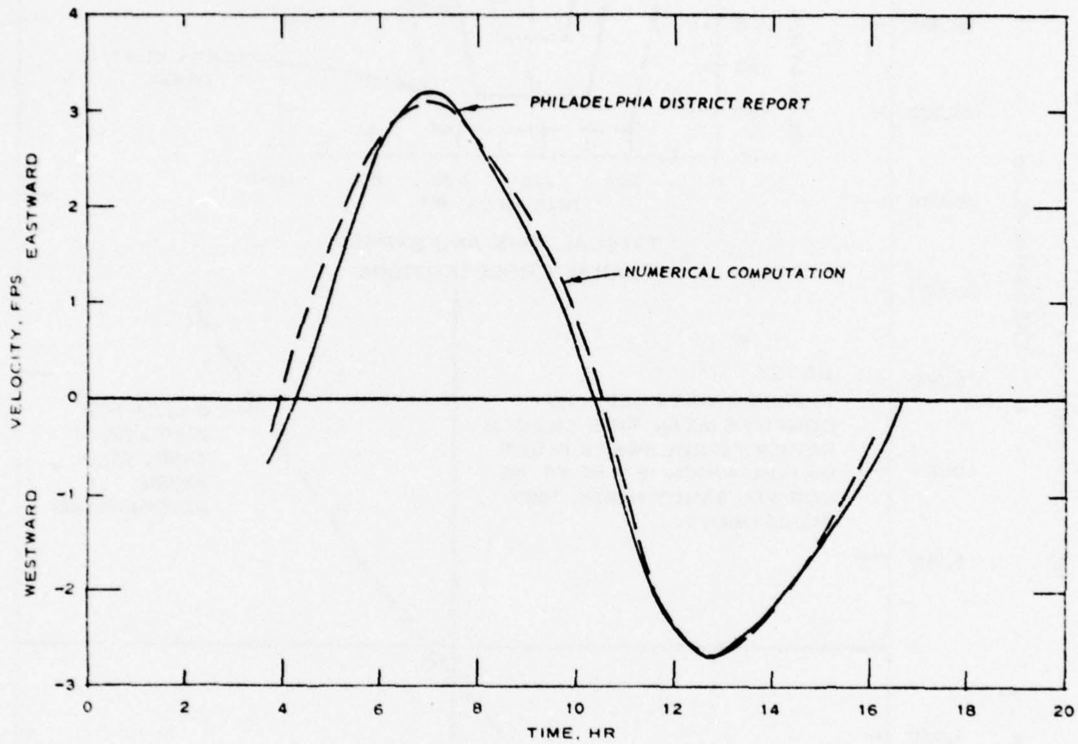

Fig. 6. Mean tide velocity curves at Chesapeake city bridge

TEST	BYPASS LENGTH	BYPASS SECTION	TIDE CONDITIONS
3	1.7 miles	15' x 150'	Extreme
4	1.0 miles	15' x 150'	Extreme
5	0.50 miles	15' x 150'	Extreme
6	0.50 miles	15' x 150'	Shifted Extreme
7	0.50 miles	15' x 150'	Shifted Mean
8	0.50 miles	15' x 200'	Shifted Extreme
9	1.0 miles	18' x 200'	Shifted Extreme

13. As indicated above, nearly all test cases were run with extreme tide conditions. The reason for using extreme conditions instead of the mean tide curves is explained below. Fig. 7 contains plots of the velocity at Chesapeake City for both mean and extreme tide conditions with no flow control in the canal. Note that the fraction of the tidal cycle during which the velocity exceeds 1.0 fps is about the same for both tide conditions. Furthermore, results from the first two test cases listed above revealed that the maximum velocity computed at Chesapeake City using the extreme tide conditions was only slightly higher than when employing the mean conditions. However, a large difference exists between the net discharges calculated for the two cases. Using the trapezoidal rule to determine the net volume under the discharge hydrographs over a tidal cycle and then dividing by the tidal cycle, the net discharge for test 1 was found to be 530 cfs, whereas, for test 2 the net discharge was 4635 cfs. From fig. 5, the net discharges for the mean (ΔH=0.2) and the extreme (ΔH=1.0) tides were approximately 1,000 cfs and 11,000 cfs, respectively, in the 27-ft by 250-ft channel. The net discharges for the 35-ft by 450-ft channel with no flow control, for the mean and

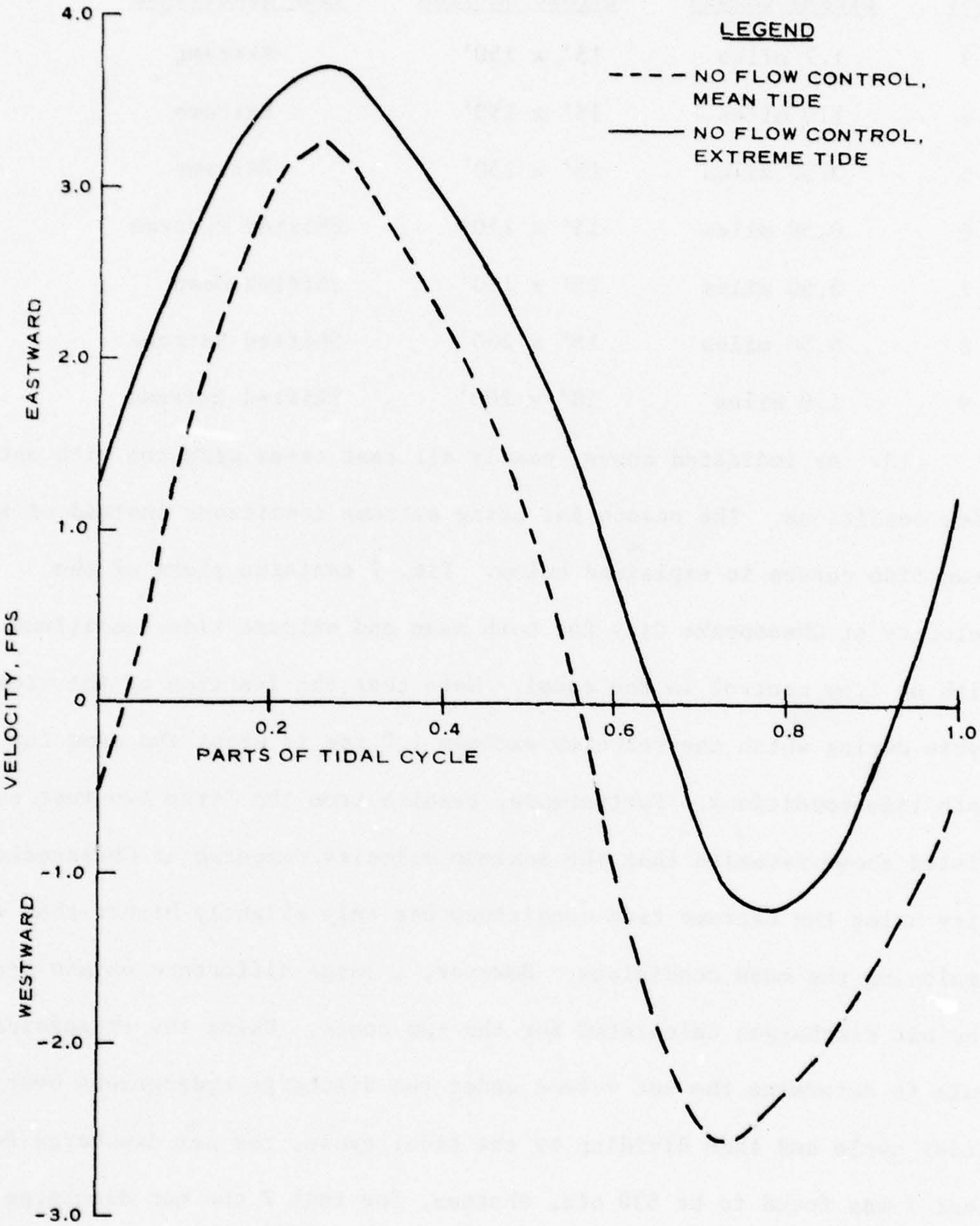

Fig. 7. Velocities at Chesapeake City

extreme tides, were determined to be 5,460 cfs and 28,800 cfs, respectively. Since the fraction of the tidal cycle during which the velocity exceeds 1.0 fps and the maximum velocity at Chesapeake City are about the same for both tide conditions, whereas, net discharges are an order of magnitude higher for the extreme tide conditions, the decision was made to use the extreme tide curves. It should be noted that as a result of the first two tests it was obvious that the criterion that net discharge should be reduced to values comparable to those that occurred in the 27-ft channel could easily be satisfied.

14. Tests 3, 4, and 5 consisted of an attempt to satisfy both the discharge and velocity criteria by reducing the bypass canal length while holding the cross section constant, i.e., 15 ft by 150 ft. Reducing the bypass length resulted in an increase in the net discharge; however, even with the length reduced to 0.50 mile, the criterion that the net Q be less than 11,000 cfs is still satisfied. The effect on the velocity in the main canal as a result of reducing the bypass length was to increase the maximum value as well as to increase the total time over a tidal cycle when the velocity is above 1.0 fps. These results are illustrated in fig. 8. Note that for test 5, i.e., bypass length = 0.50 mile, the net discharge is about 10,000 cfs and the velocity at Chesapeake City in the main canal exceeds 1.0 fps about 0.55 of the tidal cycle. As previously noted, from fig. 6 it can be seen that the velocity exceeds 1.0 fps about 0.70 of the tidal cycle for the 27-ft by 250-ft main canal.

15. After the tests discussed above were run, the results from the physical model test indicated that the tide curve at Courthouse Point should have a forward phase shift of about 30 minutes. The results from

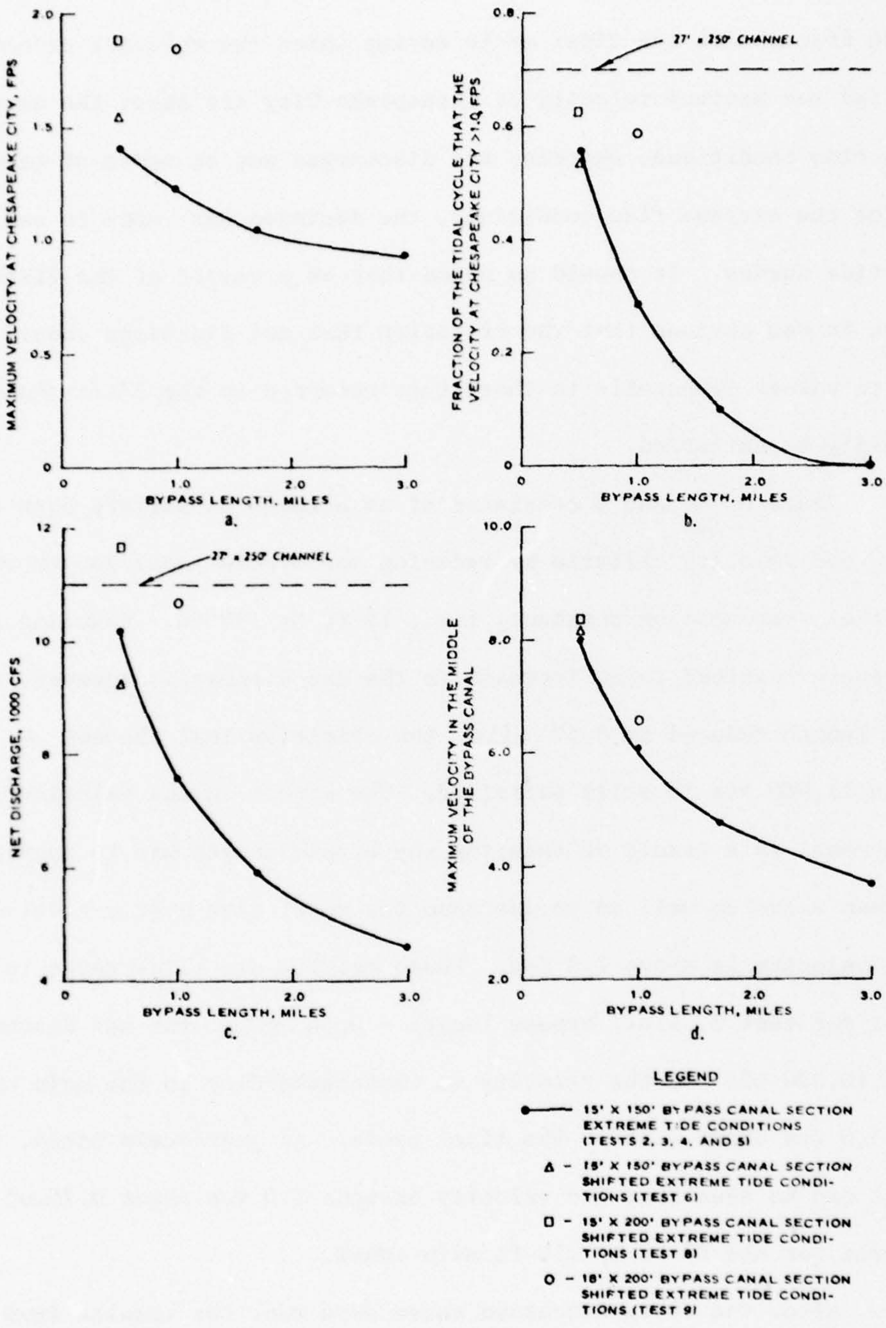

Fig. 8. Presentation of results from numerical tests of flow control plan

test 6 (see fig. 8) indicated that the effect of such a shift was to decrease the net discharge, increase the maximum velocity at Chesapeake City, and slightly decrease the time over the tidal cycle when the velocity exceeds 1.0 fps. All further tests were run with the shifted tide conditions. Test 7 was the same as 6 except the shifted mean tide conditions were employed. From fig. 9 it can be seen that the velocity remains above 1.0 fps for about the same fraction of the tidal cycle for both the shifted mean tide conditions and the shifted extreme conditions.

16. Up until now only two criteria which the flow control scheme must satisfy have been discussed, the reason being that these were the only two criteria initially imposed by the Philadelphia District. However, as the net flow and velocity criteria were approached, velocities in the bypass canal became extremely high. Fig. 10 contains plots of the velocity in the middle of the bypass canal for tests 6 and 7. As can be seen, the maximum velocity is over 8.0 fps for the extreme shifted tide conditions. In order to maintain navigable conditions in the bypass canal for small pleasure craft, the maximum velocities probably should not exceed 6.0 fps for an appreciable portion of the tidal cycle. From the plots shown, it can be seen that for tests 6 and 7 the velocity in the bypass canal remains above 6.0 fps for over 30 percent of the tidal cycle. From the discussion above, one can conclude that a third criterion has been placed upon the flow control scheme; namely, the velocity in the bypass canal should not exceed 6.0 fps.

17. With a third criterion to be satisfied, additional tests were needed. Test 8 was run to determine the influence upon the various criteria

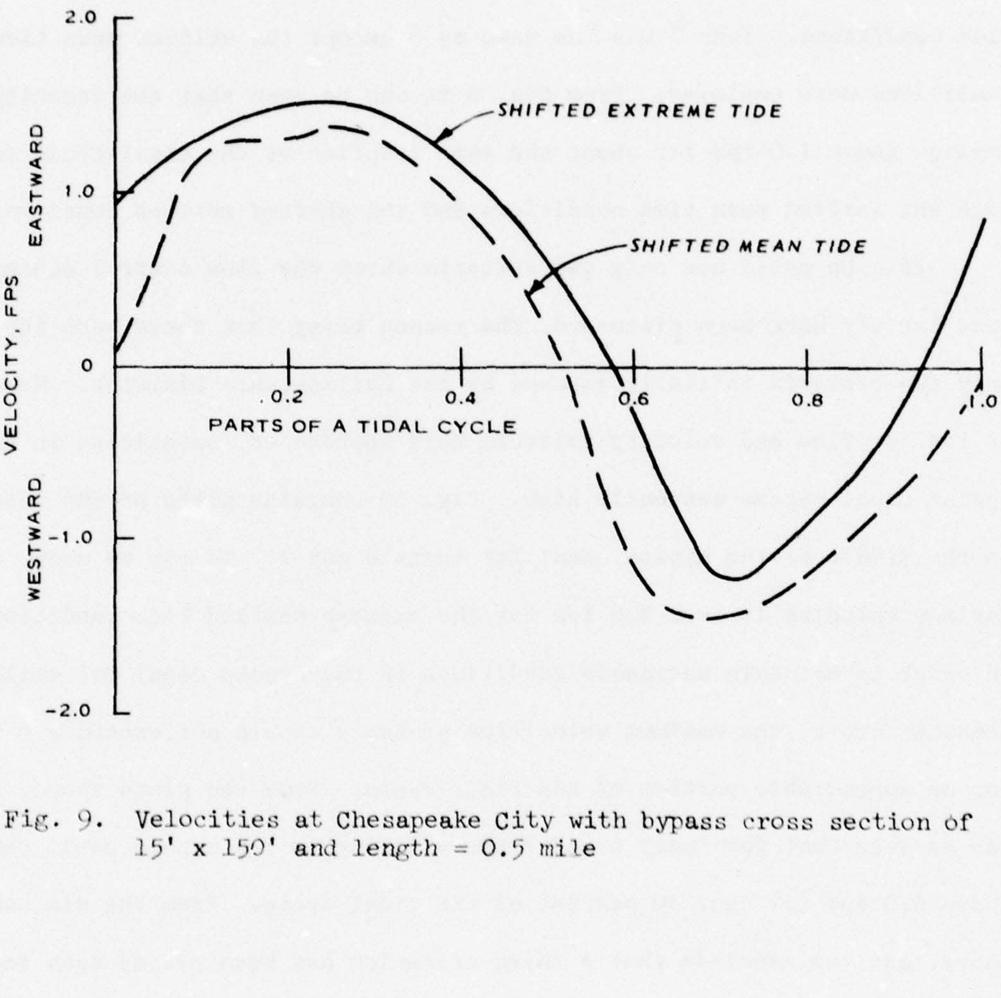

Fig. 9. Velocities at Chesapeake City with bypass cross section of 15' x 150' and length = 0.5 mile

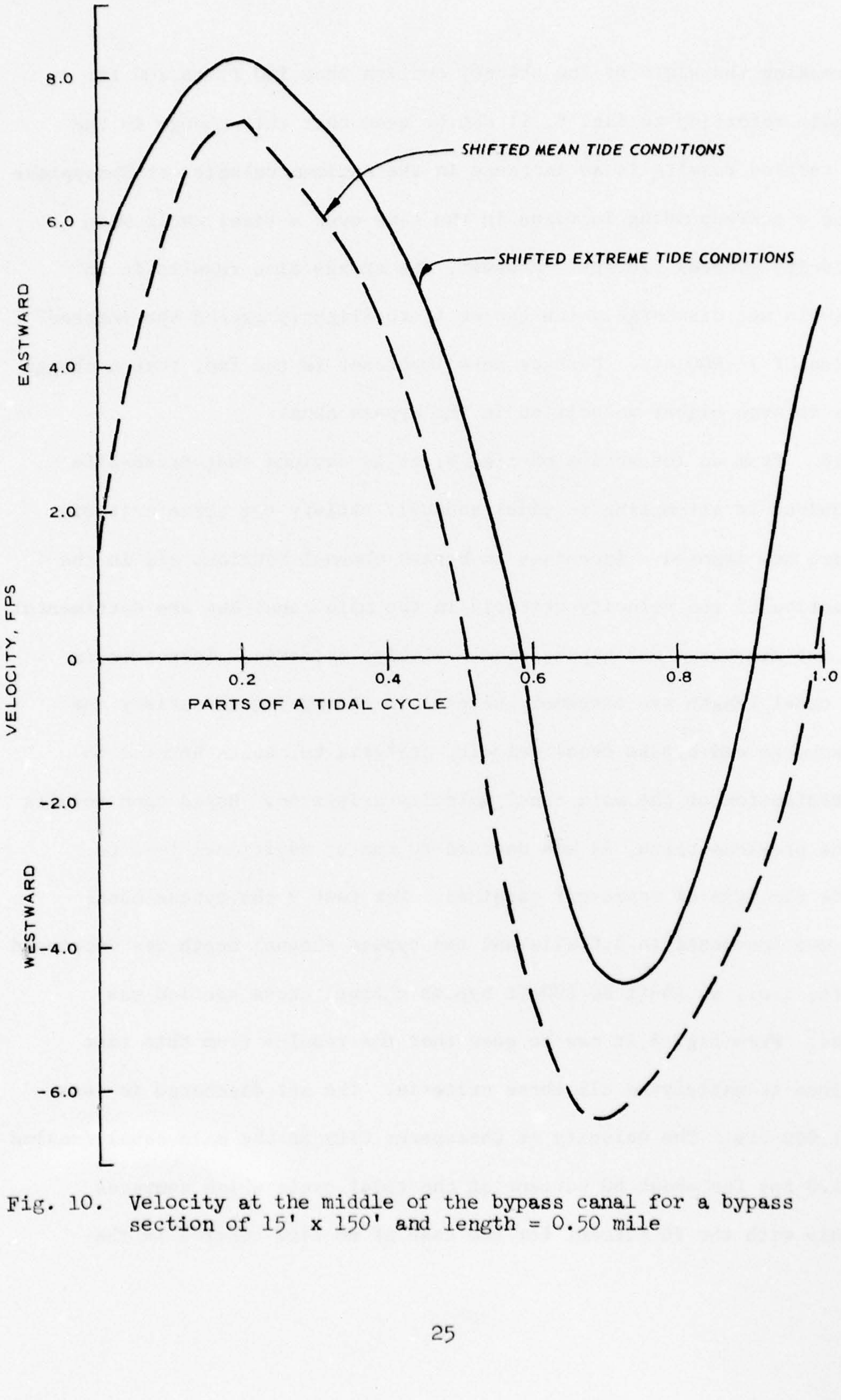

Fig. 10. Velocity at the middle of the bypass canal for a bypass section of 15' x 150' and length = 0.50 mile

of increasing the width of the channel section from 150 ft to 200 ft. Once again referring to fig. 8, it can be seen that this change in the bypass section results in an increase in the maximum velocity at Chesapeake City and a corresponding increase in the time over a tidal cycle when the velocity exceeds 1.0 fps. However, the change also results in an increase in net discharge which causes it to slightly exceed the imposed criterion of 11,000 cfs. Perhaps more important is the fact that a change results in even higher velocities in the bypass canal.

18. From an inspection of fig. 8, it is obvious that trade-offs are required in attempting to simultaneously satisfy the three criteria which are now imposed. Increases in bypass channel sections aid in the satisfaction of the velocity criteria in the main canal but are detrimental to the net discharge and bypass canal velocity criteria. Increases in bypass canal length are extremely helpful in attempting to satisfy the net discharge and bypass canal velocity criteria but quite harmful to the satisfaction of the main canal velocity criterion. Based upon results from the previous tests, it was decided to run an additional test to indicate the type of trade-off required. For test 9 the bypass canal length was increased to 1.0 mile and the bypass channel depth was increased to 18 ft, i.e., an 18-ft by 200-ft bypass channel cross section was employed. From fig. 8 it can be seen that the results from this test came close to satisfying all three criteria. The net discharge is less than 11,000 cfs. The velocity at Chesapeake City in the main canal remains above 1.0 fps for about 60 percent of the tidal cycle which compares favorably with the 70 percent for the case of no flow control in the

27-ft by 250-ft main canal channel section. In addition, it is seen that the maximum velocity in the middle of the bypass canal is reduced to about 6.5 fps which approaches the 6.0-fps criterion imposed. Velocities in the main canal and in the bypass canal are plotted in fig. 11.

19. No additional tests for the purpose of satisfying the three criteria were run. However, one additional test for the case of keeping the bypass canal but removing the dam in the main canal was run. This run was made to determine the influence of only the bypass canal on flow in the main canal. If the dam was a gated structure, such a run would approximate the case when all gates as well as the navigation lock were open. Naturally there would be a local resistance to the flow which is neglected in this application. The results from this run indicated that the bypass canal alone causes essentially no effect on the net discharge or velocities in the main canal. With such a gated structure, essentially normal flow conditions could be maintained in the main canal until flow control was needed.

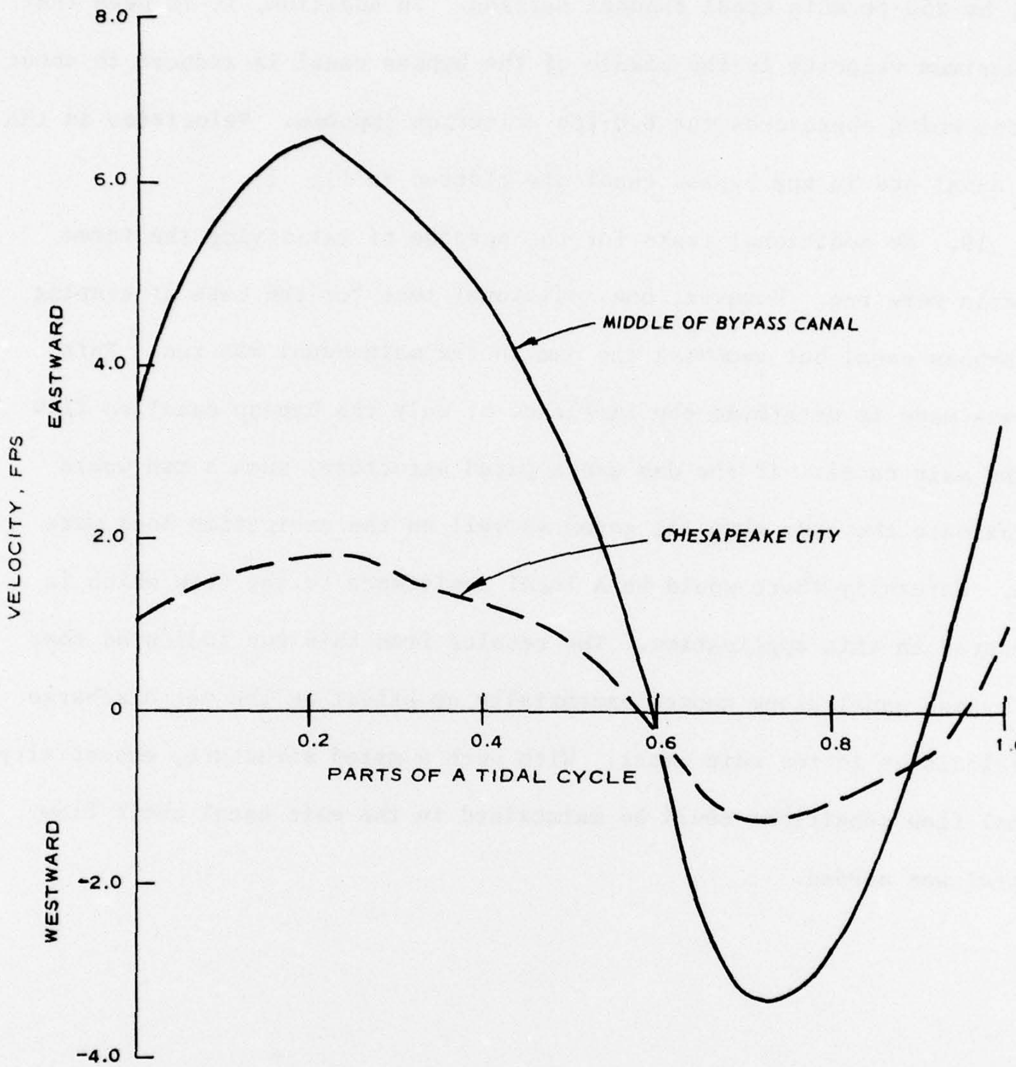

Fig. 11. Velocities taken from test 9

PART V: SUMMARY AND CONCLUSIONS

20. The project described herein was a mathematical model study to determine the feasibility of employing a flow control scheme in the C & D Canal to reduce the net flow to pre-enlargement values while maintaining velocities in the main canal above 1.0 fps over most of the tidal cycle. The flow control plan tested consisted of a navigation lock and dam in the main canal and a smaller bypass canal for the passage of pleasure craft.

21. The mathematical model employed was one previously developed by the MHD of the WES and allows for the modeling of a system containing an unlimited number of junctions and branches. The system modeled herein was composed of two junctions and five branches. The model was applied to several test cases in which different bypass canal lengths and cross sections were employed. For the first several tests, the extreme tide curves which presently occur at Courthouse Point and Reedy Point Jetty were applied. However, a test case in the physical model revealed that the flow control scheme would result in about a 30-minute forward phase shift at Courthouse Point. All further tests were then run using a shifted tide curve at the Chesapeake end of the canal.

22. The initial tests revealed that the net flow in the main canal decreased rapidly with bypass canal length as did the fraction of the tidal cycle during which velocities at Chesapeake City remained above 1.0 fps. In addition, the maximum velocity of the middle of the bypass canal also decreased rather quickly with the bypass length. The effect of employing the shifted tide conditions was to increase the maximum

velocities at Chesapeake City and at the middle of the bypass canal. However, the net flow through the main canal decreased. In addition the shifted tide appeared to result in a slight decrease of the fraction of the tidal cycle during which the velocity at Chesapeake City remained above 1.0 fps. The effect of increasing the bypass channel section was to cause an increase in the maximum velocities at Chesapeake City and in the bypass canal, as well as an increase in the net flow and the fraction of the tidal cycle during which the velocity in the main canal remains above 1.0 fps.

23. Based upon these results it was obvious that a trade-off between bypass canal length and cross section was required in order to satisfy the velocity and net flow criteria which had been imposed. A final run employing a bypass channel section of 18 ft by 200 ft and a length of 1.0 mile resulted in the three criteria almost being simultaneously satisfied. Further trade-off could have been made; however, it was felt that for the purpose of demonstrating the feasibility of such a flow control scheme these results were sufficient. Further refinement of the plan would of course be required if construction of such a plan became desirable.

24. It can be concluded that a flow control scheme consisting of a navigation lock and dam in the main canal and a smaller bypass canal can be made to satisfy the three criteria imposed, namely:

a. Net flow should be reduced to pre-enlargement values.

b. Velocities in the main canal should remain above 1.0 fps over most of the tidal cycle.

c. Velocities in the bypass canal should probably not exceed about 6.0 fps.

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3. Johnson, B. H., "Unsteady Flow Computations on the Ohio-Cumberland-Tennessee-Mississippi River Systems," to be published as a WES Technical Report.

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Table 1

SPATIAL AND TIME STEPS EMPLOYED

Bypass Length miles	Branch Spatial Steps, ft					Δt, sec
	1	2	3	4	5	
3.0	2112	1594	2640	2112	2640	44.712
1.7	2112	1056	2244	1056	2112	22.356
1.0	2112	528	1320	528	2112	14.904
0.50	2112	264	660	264	1936	7.452

APPENDIX A

POSSIBLE FLOW CONTROL PLANS

1. Introduction. During earlier phases of the Chesapeake and Delaware Canal project, several flow control plans were considered as possibilities in the event that flow control should ever become necessary.[1] Further analysis of these plans, taking into account the objectives of the flow control plan and the operational aspects of the various plans, has shown most of these plans to be impractical. As discussed in this report, the primary objective of the flow control plan is to return net flows to approximately preproject conditions (250' x 27' channel conditions). A more recent requirement of the flow control plan is to maintain canal velocities above 1 fps the majority of the time during the striped bass spawning season. The various plans which have been considered can be grouped in the following categories:

Navigation Lock and Dam

Navigation Lock and Dam with Bypass Canal

Control Structures

Restricted Waterway Section

These various plans will be briefly discussed in the following paragraphs to document the rationale for selection of the plan involving a navigation lock and dam and a bypass canal as the most practical plan.

2. Navigation Lock and Dam. This plan would provide a 110-ft by 1200-ft lock through which all canal traffic would have to be locked. The remaining portion of the dam could include control gates to provide a capability for flow control. It is conceivable that a single partial gate opening (as opposed to continuous adjustment) could be identified

that would return net flow through the canal to approximately preproject conditions and maintain canal velocities above 1 fps during the majority of the tidal cycle. However, this plan would have a very serious effect in that it would significantly slow canal traffic since all traffic would have to be passed through the lock. During periods of the year when flow control might not be needed, the lock and other control gates could be opened to allow essentially uncontrolled flow; nevertheless, traffic would be significantly slowed because of unidirectional traffic through the structure and the care needed to pass safely through the 110-ft-wide lock.

3. Navigation Lock and Dam with Bypass Canal. This plan would provide a 110-ft by 1200-ft lock in the main canal for use by deep-draft vessels but would provide an uncontrolled bypass canal of smaller dimensions for use by smaller draft vessels. It has been estimated that a high percentage of the canal traffic could be passed through a 12-ft-deep bypass canal. The dimensions and length of the bypass canal would be designed to return net flow conditions to approximately preproject conditions and to maintain main canal velocities above 1 fps during most of the tidal cycle. Thus, this plan offers several advantages over the previous plan. First, it results in less interference with navigation since only the deep-draft vessels would be required to lock through the structure. Second, during periods of the year when flow control is desired, closure of the lock and dam will result in the desired net flow and main canal velocities (i.e., no gate opening adjustments required). During periods of the year when flow control might not be needed, opening the lock and control gates in the dam would result in essentially uncontrolled flow in the 450-ft x 35-ft canal. During these times shallow-

draft vessels could use either the bypass canal or the open lock while deep draft vessels would navigate through the open 110-ft-wide lock.

4. Control Structures. Several types of control structures were considered including plans involving a vertical lift gate, a horizontal sliding gate, a controlled weir section with 200-ft-wide inflatable fabric barrier, and inflatable fabric barriers across the entire channel. Each of these plans would involve complete blockage of the canal when the structure was closed and essentially uncontrolled flow at other times. The structure must be in the open position to pass any type of vessel. Therefore, during periods when flow control is desired, it would be necessary to open and close the structure at frequent intervals. Estimates of operation time for opening the structure range from about 20 minutes for the vertical lift gate to about 1 hour for the fabridam.

5. Several aspects of these plans make it unlikely that effective flow control would be possible. Flow in the canal is controlled by very long period (tidal) waves and net flow is usually the difference between large mass flux in each direction. A few simple calculations will be used to show the order of magnitude of the change in the flow situation required to reduce the net eastward flow by about 4000 cfs. To accomplish this, the volume of eastward flow would have to be reduced as shown below.

$$\text{Volume reduction} \approx 4000 \times 12.42 \times 60 \times 60 \approx 179{,}000{,}000 \text{ ft.}^3$$

For mean tide conditions, the uncontrolled eastward flow varies from 0 to about 60,000 cfs and back to 0 over about 6.5 hours. Hopothetically, to reduce the net eastward flow by the desired amount would require the blockage

of eastward flow for about 0.8 hour at the peak discharge rate or proportionately longer at lesser discharge rates. The point being made is that a major change in the flow situation is required to reduce the net flow to preproject values. It is also obvious that during any closure a head differential will develop across the structure resulting in an abnormal flow situation when the structure is reopened. This effect would have to be compensated for in any operational plan.

6. To further complicate an operational plan involving intermittent structure closure, the flow situation in the canal is extremely variable from tidal cycle to tidal cycle. Therefore, unless controlling tides at the canal extremities were being monitored and used for extensive real-time calculations, intermittent barrier closures would be completely ineffective. Even if an elaborate monitoring and control system were provided, ship traffic would interfere with barrier use at opportune times and further complicate effective flow control.

7. Reduced Waterway Section. This plan would involve the use of groins or deflection dikes to induce greater head losses during eastward flow. To effect an appreciable reduction in net flow, the system of groins would have to extend over an extensive length of canal. This would result in a restricted canal width over much of the C & D Canal, thus greatly reducing the benefits of the canal enlargement project.

8. Conclusions. Consideration of the various plans discussed above led to the decision that the most practical plan was the plan involving a lock and dam in the main canal with a bypass canal of smaller dimensions. This plan provides the desired flow control without the need for operation of a control structure. Approximate preproject net

flow conditions are reestablished through the additional resistance losses induced by the bypass canal. The plan will result in some delay for deep-draft vessels using the canal but it is believed that such delay would be less than for any other plan which provides effective flow control.

APPENDIX B

NOTATION

A	Cross-sectional flow area
B	Effective width of water surface
g	Acceleration due to gravity
h	Water-surface elevation above mean sea level
n	Manning's resistance coefficient
q	Lateral inflow per unit distance along the channel
R	Hydraulic radius
t	Time
V	Mean flow velocity
x	Distance along the channel
Δt	Time increment
Δx	Distance increment
$\partial/_{\partial t}$	Rate of change with respect to time
$\partial/_{\partial x}$	Rate of change with respect to distance

In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below.

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